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Experimental evaluation of the common defects in the execution of reinforced concrete beams under flexural loading

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KEYWORDS

Concrete;
Deficient RC beams;
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Abstract Design of any structural element should realize the appropriate load capacity to serve the purpose of construction beside the esthetical function. Therefore, the accompanied symptoms of distress during loading conditions like cracking, deflections, and strain distribution all over the section will definitely influence the performance of these elements and their durability in sequence. Flexural moment is the most dominant straining action in many of the reinforced concrete elements such as beams, slabs, and frames. Thus, in this investigation an experimental program was carried out on deficient concrete beams which were somewhat designated to simulate the possible defects in the field, like errors in the arrangement of main steel, splices in different places (even at the maximum moment zone). Faults of improper workmanship were represented using a beam of honey-combed concrete and other of insufficient cover. On the other hand, a control beam was parallely cast for the purpose of comparison. Measurements like strains of concrete and steel, deflections and propagation of cracks were all observed and detected to evaluate to how any of these practice faults influence the behavior of beams. It was found that well-arranged distribution of reinforcement improves the ductile behavior of failure and reduces the corresponding deflections. Meanwhile, eccentricity of main steel creates a sort of non-uniform stress distribution over the section and accelerates approaching failure stage. In addition, the honey-combed structure undergoes more symptoms of distress and approaches failure faster without intermediate stage. Despite the fewer grids of cracks noticed, the honey-combed beam exhibits higher deflection values.

Introduction

Nowadays, the causes of deficiency in RC elements have been widely investigated. Deficiency was found as the result of faults in design, the use of unsuitable materials, improper workmanship, and exposure to aggressive environmental conditions, excessive loading, or a combination of two or more of such

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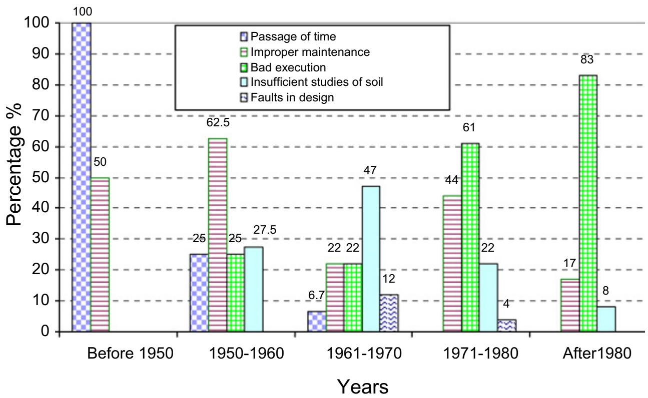

Fig. 1 Causes of deterioration according to the year of construction, Hbnrc [1].

errors. So, repairing and restoration programs are carried out to return the concrete members to a satisfactory condition of structural adequacy, durability, and appearance. If the evaluation of damage is done accurately, then a satisfactory repairing will be obtained. The Housing and Building National Research Center, HBNRC, has operated a statistical study on the causes of deterioration in concrete structures at different periods. Fig. 1 illustrates that about 83% of the causes of damage were referred to bad execution practices starting from the eighty's of last century. Thus, there is an increasing demand for developing a better understanding of the effect of bad execution practices on the performance of concrete structures especially on cracking and deflection in order to determine the proper method of repairing these defects, Fig. 1. Cracking at service loads should not be such as to spoil the appearance of member or to lead to excessive deformations. This may be achieved by specifying allowable limits on the crack width [2,3,6]. Actually, it's not the issue of the present research to state the best way to determine crack width and spacing as the present study is more concerned with the propagation of cracks, the patterns of failure and comparing them individually with an ideal beam (without defects). On the other hand insufficient lap splice length in flexural members generally results in premature, sudden brittle failures due to inadequate bond strength [4–6]. However, to the authors' best knowledge, no researches had been conducted to study the effect of the congestion of reinforcing steel bars in one side of the beam.

The present research aims at introducing a rational evaluation of the common shortcomings in the execution of RC beams under flexural stresses. Experimental program consists of eight reinforced concrete beams which were designed to simulate the most common defects that really exist in situ including honey combed concrete, insufficient concrete cover, splicing main steel even at sections of the maximum bending stresses as well as the eccentricity of reinforcing bars. In addition, one control beam was tested for the purpose of comparison. Cracking, mode of failure, load–deflection relationship, and steel strains of the main reinforcement at the splice region were recorded for all tested beams and analyzed to investigate the effect of the key parameters.

Test program

Characteristics of laboratory specimens

The experimental test program involved 9 RC beams $150 \times 300 \times 2000$ mm designated as B1 to B9. The program comprised two main categories of beams as follows:

Defects of the reinforcing steel

This category contains 6 beams and classified into two series, eccentricity of main steel bars and splices, Table 1. The first series contains two beams, B2 and B3, through which the main reinforcement was congested at one side. B2 was made with two congested reinforcing bars and one individual while the other beam B3 had its whole reinforcement congested at one side. For the second series, four beams (B4–B7) were tested to investigate the harmful effect of bad splicing with different configurations. Splicing main steel was of the adjacent type as no space left between spliced bars. Either one third or two thirds of the total reinforcement spliced at the middle section or thirds with splice length (30 cm). This length is shorter than the minimum and recommended development length due to the ECP203–2007 [7].

Defects of the concrete

The common defects of concrete, like honey combing and insufficient cover, were represented by two beams B8, B9 respectively.

All the previous beams were tested and referred to the control beam B1 which was made with the standard requirements of good compacting, enough concrete cover, well-arranged reinforcement as well as no splices were used, see Table 1. All beams were constructed in the laboratory of the Housing and Building National Research Center and tested under

Table 1 Details of the tested beams.

Beam No.	Defect	No. of splices	Lap splice position	Notes
B1	Control beam	–	–	–
B2	Congested main rft.	–	–	Two congested bars
B3	Congested main rft.	–	–	Three congested bars
B4	Splicing 66% of main steel	2	Two thirds	[a]L_s = 300 mm
B5	Splicing 33%of main steel	1	Middle	[a]L_s = 300 mm
B6	Splicing 66%of main steel	2	Middle	[a]L_s = 300 mm
B7	Splicing 66%of main steel	2	The same third	[a]L_s = 300 mm
B8	Honey combed concrete	–	–	–
B9	Insufficient concrete cover	–	–	8 mm concrete cover

[a] L_s, splice length.

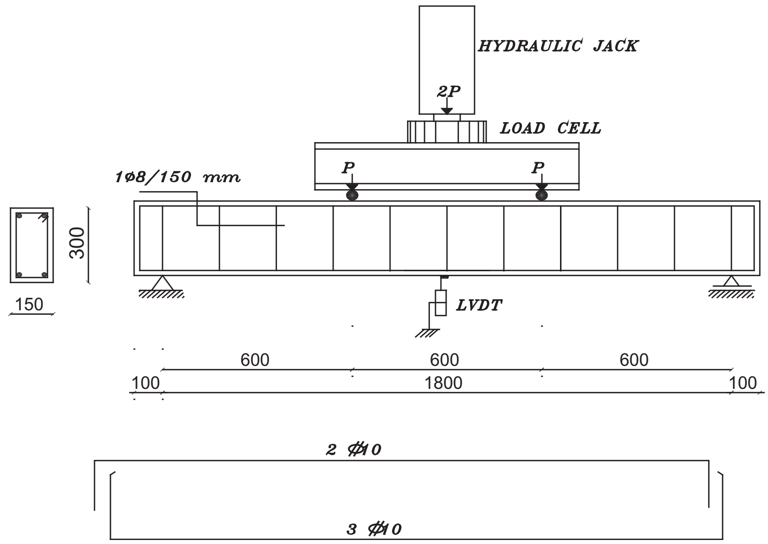

Fig. 2 Test setup and reinforcement detail of beams B1, B8, B9.

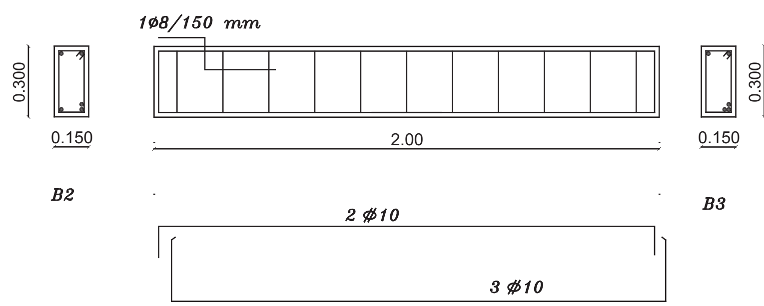

Fig. 3 Reinforcement detail of beams B2, B3.

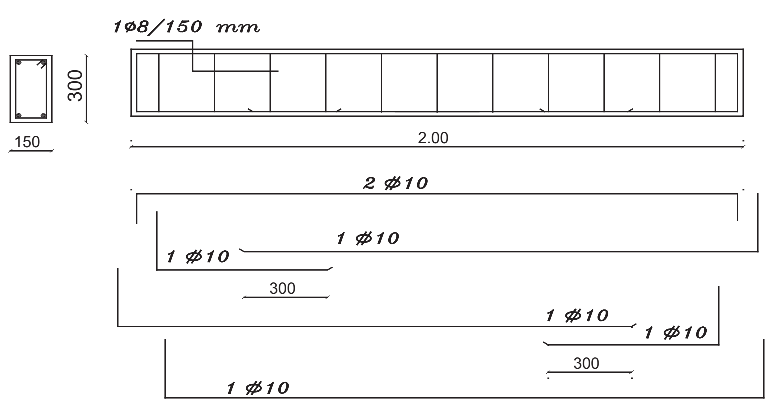

Fig. 4 Reinforcement detail of beams B4.

positive bending moment. Details of the beams are shown in Figs. 2–7.

All beams were reinforced by high grade steel 36/52 on both tension and compression sides. Three longitudinal steel bars 10 mm were used in tension side whereas two 10 mm bars were used in the compression side. To avoid shear failure, 8 mm mild steel (24/37) stirrups were provided in the shear span for all beams at spacing 150 mm. Concrete clear cover of approximately 25 mm was provided for all beams according to the ECP203–2007 except B9 which had 8 mm cover. In addition, six standard cubes $150 \times 150 \times 150$ mm were cast with the test beams as control specimens to evaluate the actual concrete compressive strength. These cubes were cured with the test beams and tested in the same day of testing the corresponding beams. All beams were fabricated with concrete of grade of 25 MPa.

Materials

The high grade reinforcing steel 36/52 used had yield stress of 430 MPa and the corresponding ultimate strength is 610 MPa. Table 2 shows the mix proportions of the concrete mix.

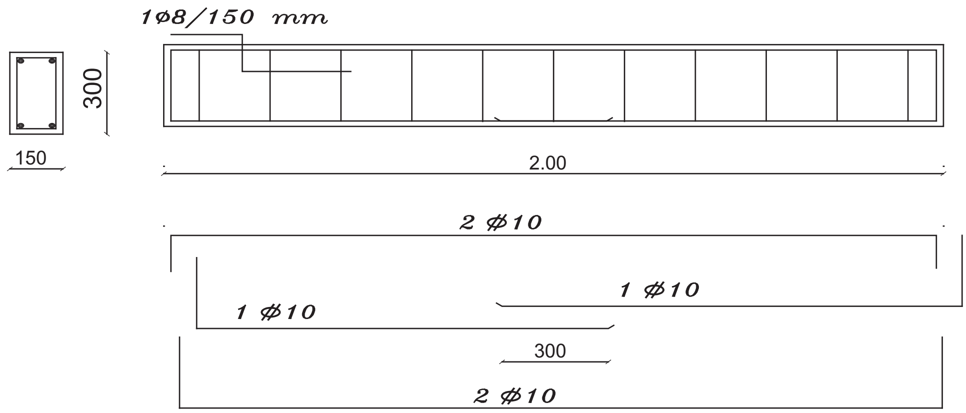

Fig. 5 Reinforcement detail of beams B5.

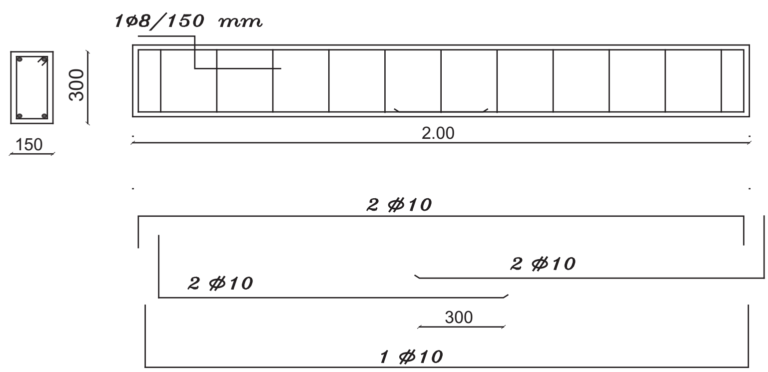

Fig. 6 Reinforcement detail of beams B6.

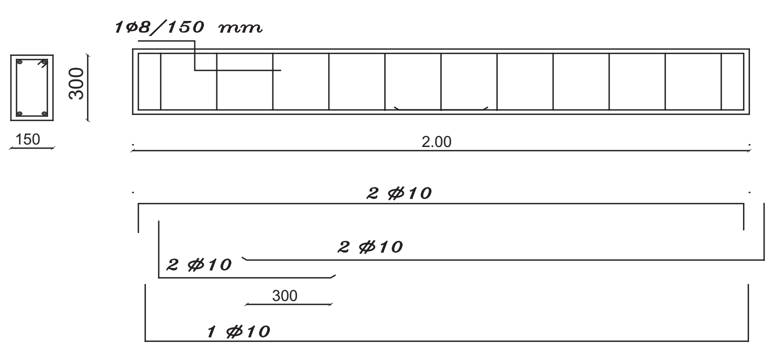

Fig. 7 Reinforcement Detail of Beams B7.

Table 2 Mix proportions of the concrete mix.

Cement (kg/m^3)	Dolomite kg/m3	Sand kg/m3	Water liter/m3	Admix. liter/m^3	Comp. strength (MPa)
250	1400	700	160	2.5	26.50

Ordinary Portland cement, siliceous sand, coarse aggregates size 10 mm, and super-plastisizer type (d), [7], were used with the quantities shown in Table 2. The average compressive strengths F_{cu} measured at the time of testing the beams were approximately 26.5 MPa.

Fabrication of tested beams

Fabrication of tested beams started with the preparation of steel bars and adjusting splices to the required lengths and positions. Then, strain gages were attached to steel bars at

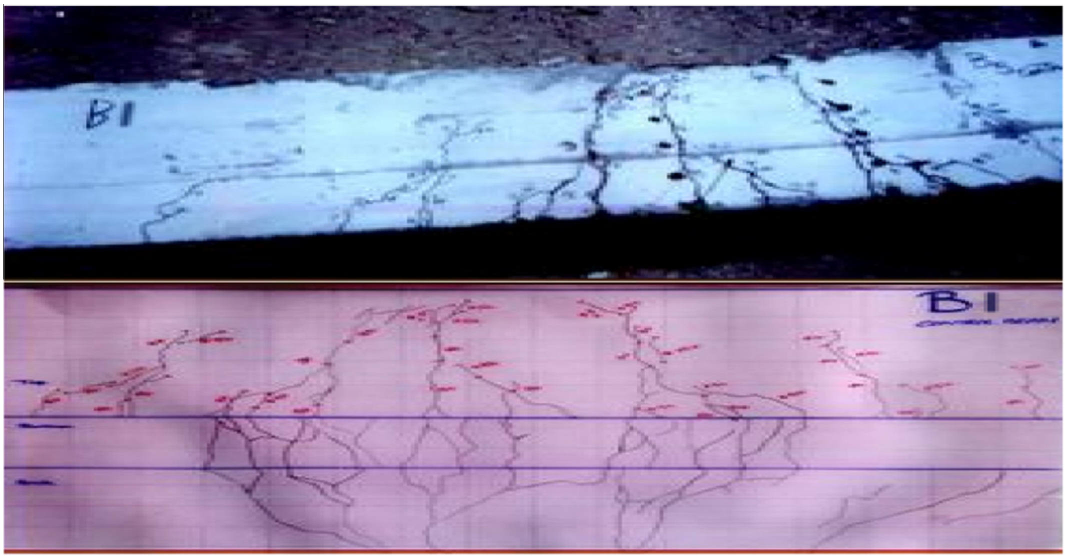

Fig. 8 Cracking pattern of beam B1.

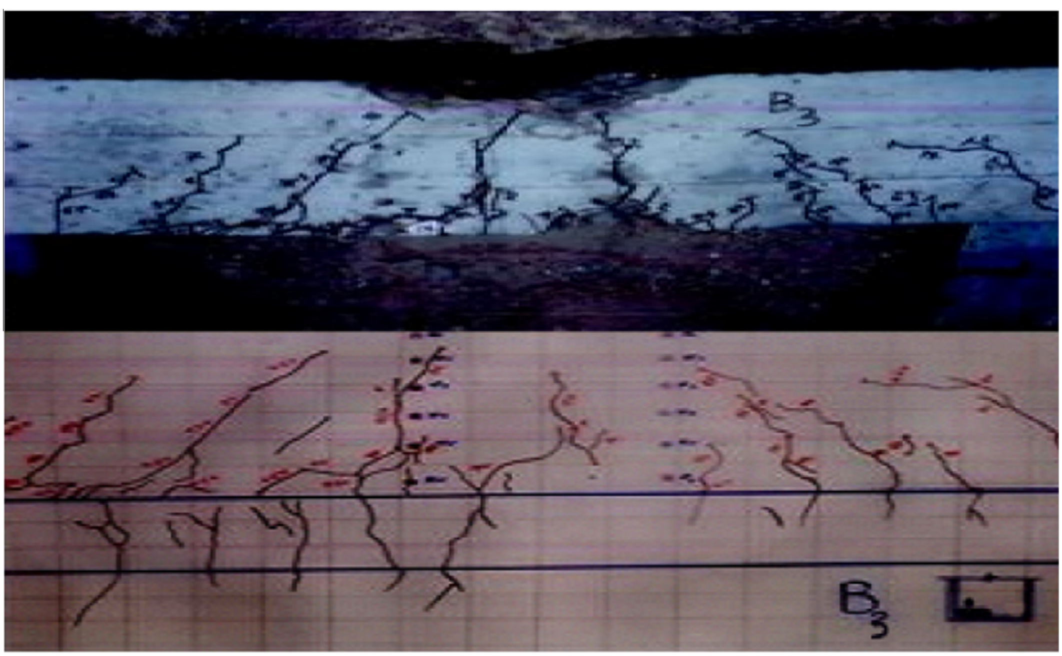

Fig. 9 Cracking pattern of beam B3.

the position of maximum deflection in a manner to assist in comparison purposes. In beams with splices there were three strain gauges. A strain gage was fixed at the outer spliced bar, the second was attached to the inner spliced bar while the third was fixed at the continued bar which always kept as the middle bar. A rotated pan mixer of $0.15\,m^3$ capacity was used in mixing concrete, which was placed in a wooden form. A mechanical vibrator was used to compact concrete, while for B8 manual compacting was used. All beams were kept in the laboratory temperature (25 °C) and covered with wet burlaps.

Test setup and instrumentation

All beams were tested using a 2000 kN double portal, open reaction frame. Load was applied using a hydraulic jack of 400 kN capacity in compression. The hydraulic jack was attached to the cross girder of the double portal frame. The jack was equipped with a tension/compression load cell of ± 680 kN capacity to measure the applied load. The load was distributed equally by a spreader beam to two points along the specimen to generate a constant moment region at midspan. Electronic strain gauges were bonded to steel reinforcement to measure

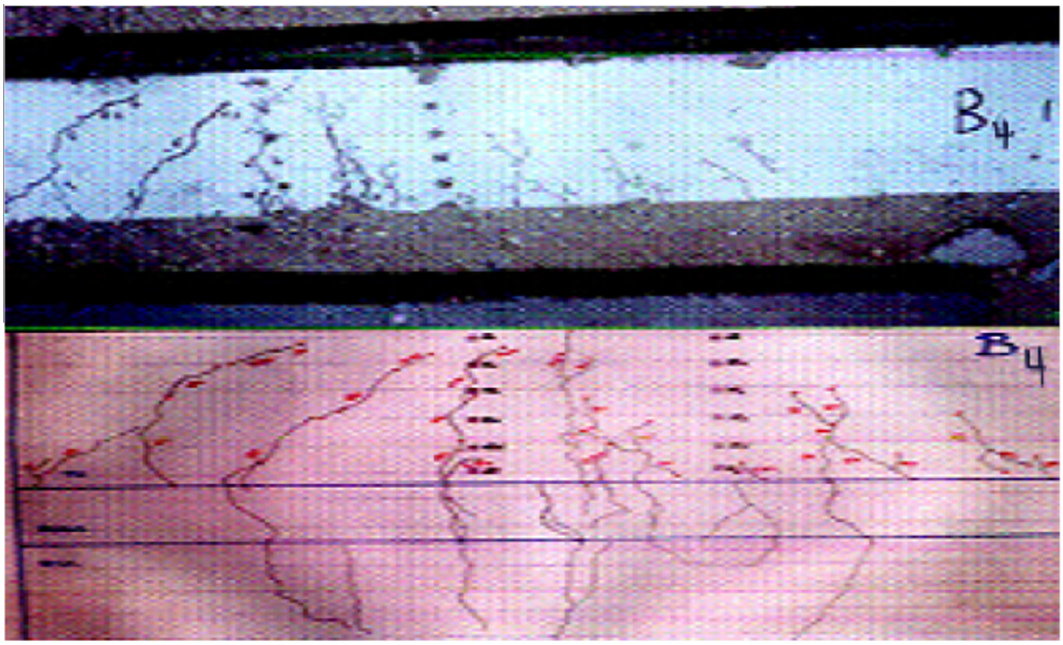

Fig. 10 Cracking pattern of beam B4.

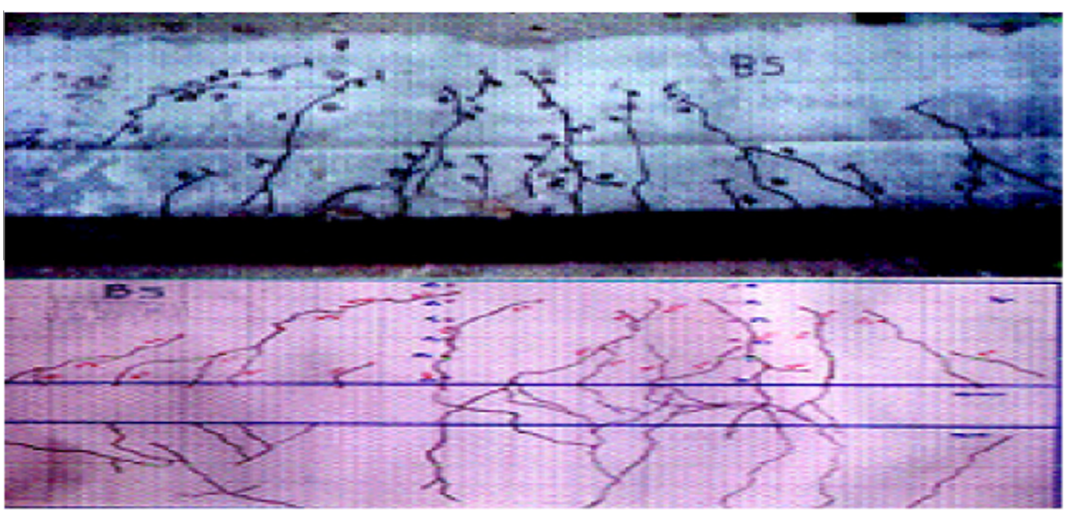

Fig. 11 Cracking pattern of beam B5.

the actual steel strain. Linear variable differential transducer (LVDT) with stroke ±50 mm and sensitivity 0.1 mm was mounted at the bottom side of the midspan for each specimen. At each load stage, the electrical strain gauges, load cells and (LVDTs) voltages were fed into the data acquisition system.

The voltage excitations were read, transformed and stored as force, micro strains, and displacement by means of a computer program that runs under the Lab View software. The cracking pattern was monitored with each load stage. The test was continued after the ultimate load in order to evaluate the post peak behavior of the tested beams. The cracking and ultimate loads were recorded. In addition, the pattern of cracks at all sides of beam was neatly sketched with scale 1:1 and photographed.

Test results and analysis

Cracking behavior and modes of failure

The test beams were loaded to failure and the observed behavior in terms of cracking, modes of failure, load–deflection response were recorded. The appearance of the tested beams after loading is shown in Figs. 8–14. The control beam B1 showed the typical cracking behavior of the under reinforced concrete simple beam and failed in ductile flexural tension. At early loading levels, vertical cracks appeared at region of maximum tensile stresses. Upon increasing the load, the number, width and extensions towards the compression zone

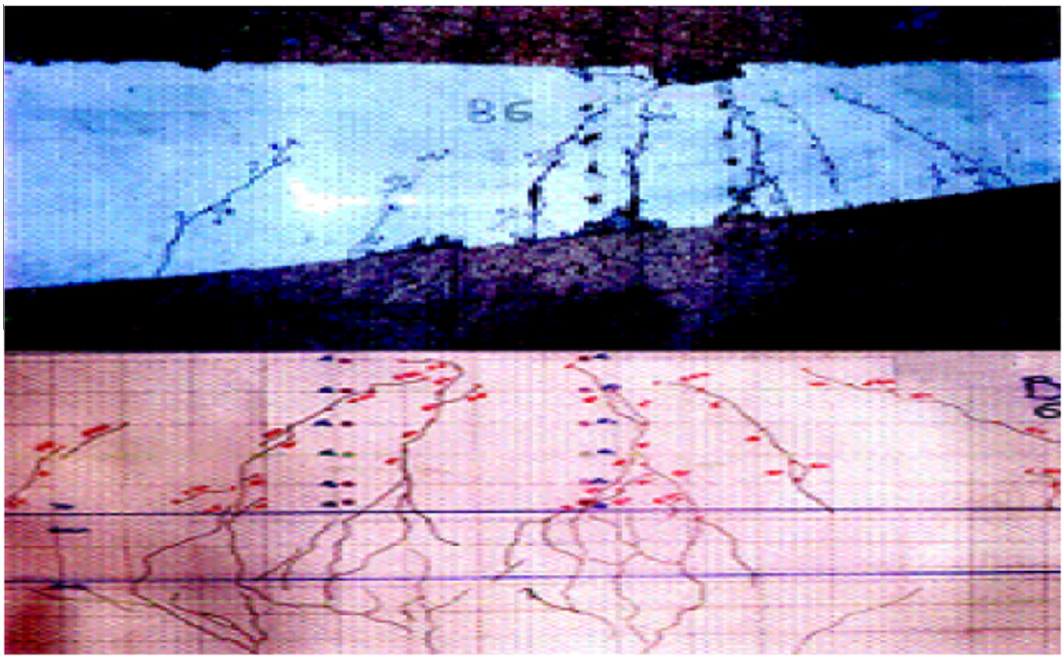

Fig. 12 Cracking pattern of beam B6.

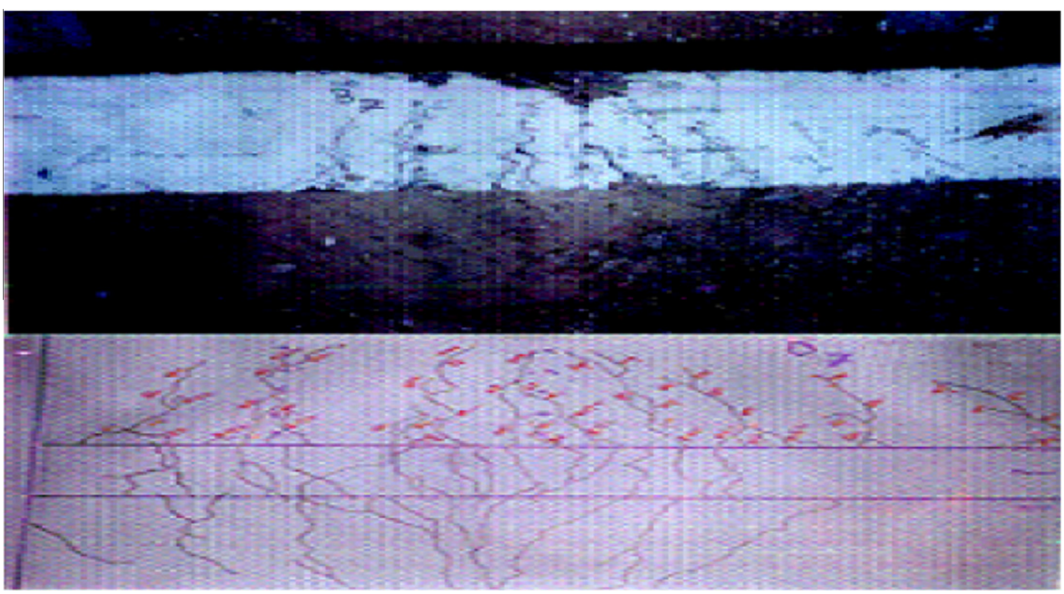

Fig. 13 Cracking pattern of beam B7.

of the cracks increased. At later stages of loading, yielding of longitudinal reinforcement occurred and the cracks rapidly grew and the beam experienced large deformations. Finally, successive concrete crushing at compression side of midspan took place. For beams with eccentric main steel, B2 and B3, the point of the maximum tensile stress was that edge of beam where the main reinforcement was congested. Certainly, increasing the percentage of eccentricity increased the difference in the applied stresses and the sequent induced strains between the two beam sides. The pattern of cracks indicates a dissimilar appearance on the two beam sides, Fig. 9. They were wider, denser and highly propagated at the side of the congested reinforcement and this may ascertain the state of non-uniform stress distribution. However, the mode of failure of beams B2, B3 can be considered as ductile one.

For B4, with inadequate lap splice length at the two staggered thirds of the span, the rate of losing flexural rigidity was close to control beam approaching the sequent stages of load deflection relation occurred at the same levels of the control beam. Thus, it was noticed that splicing 2/3 of the main reinforcement at the staggered two thirds (B4) had no significant influence on the behavior of beam. It is only different that the propagation of cracks appointed to the existence of splices with the progress of loading. Comparing with B1, cracks become closer, denser and more intersecting at the latest loading levels. However, the observation of sub-tail cracks existed

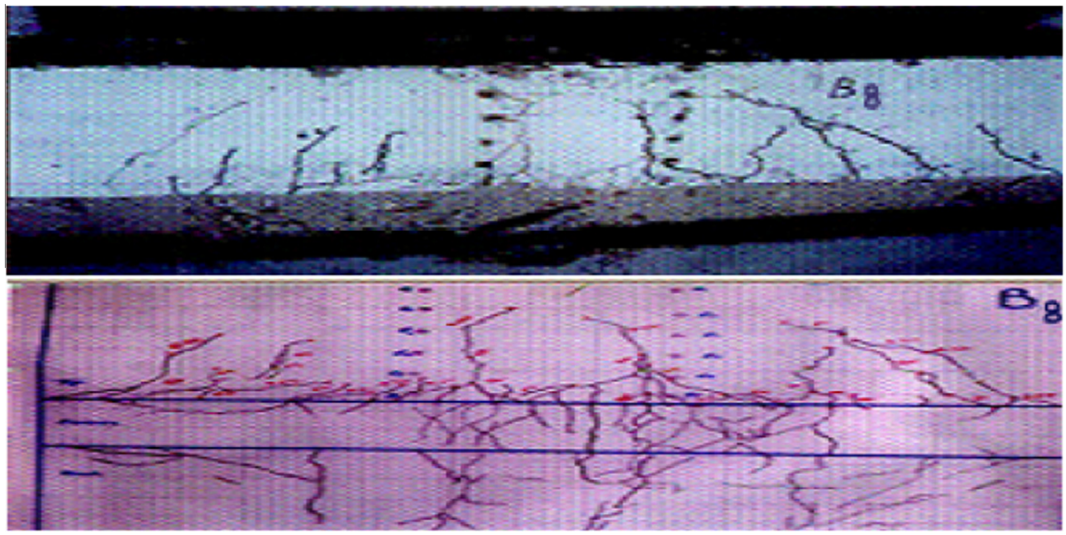

Fig. 14 Cracking pattern of beam B8.

heavier at the spliced third of B7, Fig. 13, indicating that the effect of steel bars discontinuity decreases with the distribution of splices at different positions, i.e., the rate of deterioration caused by splicing main steel at the two staggered thirds is lower than splicing at the same third. The mode of failure of beams B4, B7 can be considered as ductile one. Beams B5 and B6 with inadequate lap splice length at the middle third of the span experienced a semi ductile behavior. Two major flexural cracks initially appeared at the critical sections adjacent to the end of the lap splice zone and extended vertically by the increase of load followed by a sudden noticeable drop in the applied load combined with the formation of longitudinal splitting cracks which were initiated in the bottom cover of the tension side indicating a bond failure of the spliced steel bars. Then, the applied load started to increase again with increasing the deflection showing a ductile yield plateau due to the continuous non spliced steel bars. On the other hand, It was noticed that B8 with honey combed concrete had an earlier approach to failure at earlier loading level as well as no intermediate stage could be distinguished. Tensile stresses developed faster in concrete and when cracks initiated they propagated easily in between pores of concrete. The presence of these pores in the honey combed structure helps the propagation of cracks with more easiness. After failure, B8 was not capable of resisting any further load. The failure of B8 could be classified as pronounced sudden and brittle mode of failure. Based on test observations, it could be said that honey combed concrete presented one of the most affecting defects on the behavior of RC beams. Examining the behavior of B9 with insufficient concrete cover, it was clear that the three stages of loading can be explicitly distinguished for this beam. The beam approached the same loading levels of control beam with more or less the same deflections. However, the pattern of cracks indicated that the width of cracks reduced with the low concrete cover.

Load–deflection behavior

The total applied load was plotted against the vertical deflection measured at midspan for all tested beams as shown in

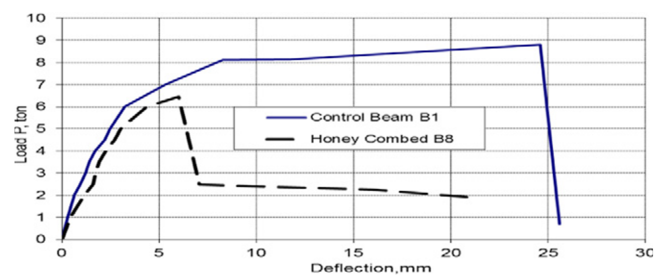

Fig. 15 Effect of honey combed concrete.

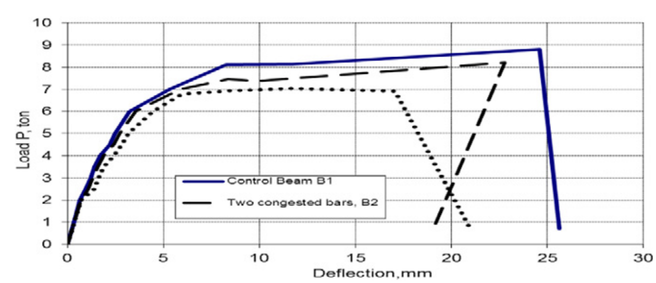

Fig. 16 Effect of eccentricity of the main steel reinforcement.

Figs. 15–19. For beams B1, B2 and B3, lower values of the ultimate load and deflection of the two defected beams B2, B3 were noticed compared with the control beam B1. The load deflection relationship for beam B9, shown in Fig. 17, leads to the same observations. On the other hand, a noticeable decrease in the ultimate load and deflection of the honey combed beam B8 compared with the control beams B1 was observed. Meanwhile, B4, B7 had almost the same load–deflection relation compared with B1. Specimens B5, B6 reached approximately the same deflections of B1 with a clear decrease of the ultimate load, Fig. 19. Ultimate load Pu, deflection at failure Δf, defection at yield Δy, steel strain at ultimate load εu, and ductility index μΔ for all the tested beams are shown in Table 3.

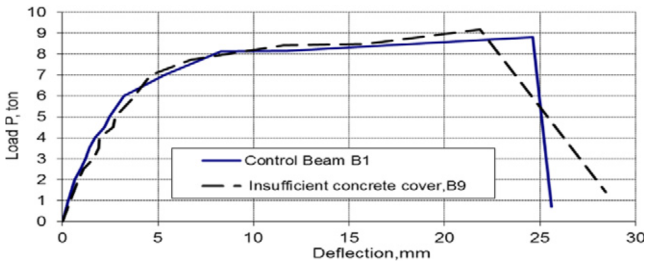

Fig. 17 Effect of insufficient concrete cover.

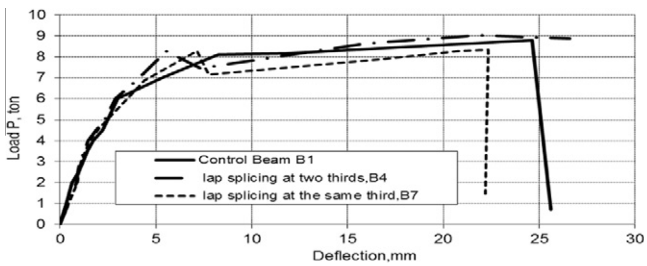

Fig. 18 Effect of splicing main steel reinforcement at the two thirds.

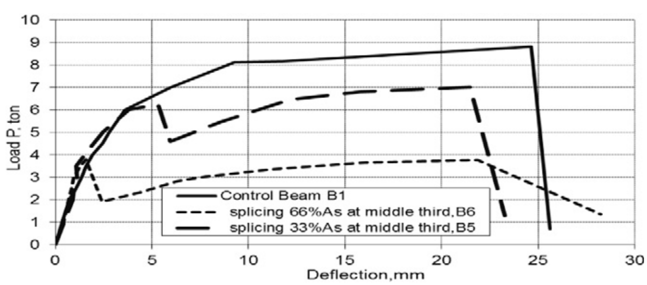

Fig. 19 Effect of splicing a percentage of the main steel reinforcement at the middle third.

Stiffness

The stiffness of each beam was evaluated as the slope of the linear ascending part of the load deflection curve and presented in Table 3. It can be obviously noticed that the defected beams (B2, B3, B4, B7, and B9) had approximately the same stiffness compared with the control beam B1. However, B5, B6 showed a little increase in the initial stiffness and this may be attributed to the increase of steel reinforcement area due to lap splices in the middle third of specimens. The initial stiffness of B5 and B6 was 14% and 12% higher than that of the control beam B1 respectively. On the other hand, the honey combed specimen B8 had relatively lower stiffness compared with B1 and this may be due to the early initiation of cracks in between pores of concrete. The initial stiffness of B8 was 21% lower than that of the control beam B1.

Ductility

Ductility is the ability of a RC member to sustain related large inelastic deformation without a major reduction in load resisting capacity. Many authors adopted the displacement ductility index, μ_Δ, to evaluate the ductility level of RC beams. However, this index, μ_Δ, was used in the current study to calculate ductility of the tested beams using Equation 1 and as presented in Table 3.

$$\text{Displacement ductility index}, \mu\Delta = \Delta_f/\Delta_y \qquad (1)$$

where, Δ_f = deflection at 80% of the ultimate load on the descending branch of the load–deflection curve, Δ_y = deflection at yield load calculated from the load–deflection curve as the corresponding displacement of the intersection of the secant stiffness at a load value of 80% of the ultimate lateral load and the tangent at the ultimate load. Referring to Table 3, the honey combed concrete had the most harmful effect on the ductility index as B8 presented the lower value of 1.35. On the other hand, insufficient concrete cover had no significant effect on the ductility index. Although B5, B6 experienced a semi ductile behavior, but both of them showed a reasonable ductility index compared with the control beam B1.

Strains of longitudinal steel reinforcement

Examining Table 3, strain gages on the longitudinal reinforcing steel bars of most of the tested beams recorded tension steel strain values in the range of 2000–4600 μ_s at the ultimate load level which indicate that the longitudinal reinforcement developed yielding, where μ_s means micro strain = strain $\times 10^{-6}$. The steel strain values for all tested beams, except for B8, are much higher than the yield strain ε_y = 2000 μ_s indicating

Table 3 Experimental results of the tested beams.

Beam	Ultimate load, P_u (ton)	Δ_f mm	Δ_y mm	Steel strain at ultimate load, $\varepsilon_u \times 10^{-6}$	Stiffness kg/mm	Ductility index $\mu_\Delta = \Delta_f/\Delta_y$
B1	8.81	24.6	7.2	4600	19100	3.44
B2	8.54	22.8	7.2	4100	18600	3.16
B3	7.05	19.1	6.9	2500	18140	2.72
B4	8.75	26.6	7.8	4300	19450	3.41
B5	7.12	21.4	4.8	1850[a], 4800[b]	21400	4.43
B6	3.65	22.1	5.1	1700[a], 5050[b]	21850	4.31
B7	8.11	21.2	6.8	4250	18900	3.11
B8	6.42	9.5	7.3	1700	15100	1.35
B9	9.18	21.9	7.15	3700	18800	3.12

[a] Strain gage record of the spliced bar.
[b] Strain gage record of the continuous bar.

that the longitudinal reinforcing steel of these beams reached the yielding range leading to ductile behavior. As shown in Table 3, the maximum strain of the longitudinal reinforcement of B8 hardly reached 1700 μ_s indicating that the honey combed concrete did not allow steel bars along the beam to participate effectively in the stress transfer.

Effect of key parameters

The main parameters included in the study were the honey combed concrete, insufficient concrete cover, splicing main steel even at sections of the maximum bending stresses as well as the eccentricity of reinforcing bars.

Effect of honey-combing

Fig. 15 and Table 3 show test results of beam B1 and B8. Both beams are identical in all terms like concrete compressive strength, reinforcement ratio and only the compaction of concrete is different. The ultimate load of B8 with honey combed concrete was 22% lower than that of the control beam B1. In addition, the displacement ductility index of B8 was decreased by 60.75% compared with the ductility index of the control beam. Among the tested specimens, B8 recorded the lowest value of P_u and μ_Δ. Moreover, the maximum recorded strain of the longitudinal reinforcement of B8 did not reach the yield value implying a poor ductile behavior.

Effect of eccentricity of the main steel reinforcement

Fig. 16 shows the load deflection relation of beams B1, B2 and B3 where B2 and B3 represent the defected beams. B2 had 66% of its main steel reinforcement congested together while B3 had 100% of the same steel reinforcement congested. The ultimate load of the defected beams B2 and B3 were lower than that of the control beam B1 by 3.5% and 19%, respectively. Table 3 indicates that the three beams had almost the same initial stiffness. Both beams B2 and B3 had post peak ductile behavior. However the displacement ductility index of B2 and B3 were decreased by 8.1% and 20%,respectively compared with the ductility index of the control beam. Referring to steel strain, the strain values of both beams B2 and B3 were higher than the yield strain value (2000×10^{-6}). As seen in Table 3, the steel strain of beam B3 was lower than the steel strain of the control beam B1 by 45.75% whereas, the steel strain of beam B2 and control beam B1 were almost identical, 4100×10^{-6} mm/mm and $4600\text{x}10^{-6}$ mm/mm respectively. It was observed that increasing the eccentricity of the main steel leads to negative effects on the overall structural behavior of beams due to the stress concentration at one edge of the beam.

Effect of insufficient concrete cover

The effect of insufficient concrete cover for beams can be discussed by comparing the behavior of beams B1 and B9 which are identical in all aspects except for the thickness of cover. Fig. 17 presents the load deflection relation for B1 with a concrete cover of 2.5 cm and B9 which had a cover of 8 mm only. Beam B9 approached approximately the same loading levels of control beam with more or less the same deflections. However, the ultimate load of B9 was 5% higher than B1 and this may be attributed to the small increase in the lever arm as the concrete cover reduced. Referring to Table 3, the overall structural behavior of beams B1 and B9 showed that insufficient concrete cover had no significant effect on load–deflection behavior or mode of failure. However, it should be emphasized that sufficient cover is essential for the durability of structural members as it protects the steel from corrosion.

Effect of splicing main steel reinforcement at the two thirds

Fig. 18 shows the load deflection relation of beams B1, B4 and B7. Beams B4 and B7 represent the defected beams which had 66% main steel reinforcement spliced at the staggered two thirds and the same third of the span respectively. The load deflection responses of the defected beams B4 and B7 were approximately similar to B1. However, beam B7 was more negatively affected from the view point of ultimate load and ductility index. The ultimate load and the ductility index of the defected beams B7 was 8% and 11% lower than that of the control one B1, respectively while B4 recorded approximately the same values of B1. As seen in Table 3, the three beams had almost the same initial stiffness. It may be concluded that the effect of the longitudinal steel bars discontinuity decreases with the distribution of splices at different positions.

Effect of splicing a percentage of the main steel reinforcement at the middle third

Fig. 19 shows the load deflection relation of beams B1, B5 and B6 where B5 and B6 represent the defected beams. B5 and B6 had 33% and 66% of its main steel reinforcement spliced at the maximum bending moment region, respectively. At early loading levels, it was noticed that the stiffness of B5 and B6 increased as well as the cracking load, compared with B1, as a result of the increase of reinforcement area due to splices. Meanwhile, the ultimate load of the defected beams B5 and B6 were lower than that of the control one B1 by 19% and 41.5%, respectively. Referring to the continuous steel bars, the strain values of both beams B5 and B6 were higher than the yield strain value (2000×10^{-6}). On the other hand the steel strain of the spliced bars did not reach the yield value beam indicating a bond failure which resulted in the observed load deflection drop. However, the three beams had post peak ductile behavior with reasonable ductility indexes.

Conclusions

This research is mainly concerned with the evaluation of the shortcomings which may exist in the execution practices of RC beams. Within the range of the investigated parameters and properties of the materials used in this work, the following conclusions could be drawn:

1. Honey combed concrete presented one of the most serious defects on the behavior of RC beams, it caused a considerable decrease in both of the ultimate load and ductility. In addition, the load deflection flexural stiffness was negatively affected.
2. Apart from the danger of corrosion of the reinforcement, the insufficient cover caused less harmful effect than the honey combed concrete. Specimen with insufficient cover

approximately approached the same loading levels of control beam with more or less the same deflections and the observed width of cracks was lower with a little increase of the ultimate load.

3. Eccentricity of the main steel led to the reduction the ultimate load and ductility. It can be concluded that the well-arranged distribution of main reinforcement delays failure, improves the ductile behavior and reduces the corresponding values of deflection.
4. Splicing 66% of the main reinforcement at the staggered two thirds even with inadequate lap splice length had no significant influence on the behavior of beam. However, the rate of deterioration caused by splicing main steel at the two staggered thirds is lower than splicing at the same third.
5. Splicing a percentage of the main reinforcement at the middle third of the span, with inadequate lap splice length, led to a semi ductile behavior. A noticeable drop in the ultimate load was observed and this drop increased proportionally with the percentage of the spliced area of steel. Meanwhile, the ductility index did not decreased due to the existence of non spliced steel bars.

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